3.3.9 LRFD Stud Shear Connector Design for Straight Girders

For straight girders, LRFD stud shear connector design is similar to LFD shear connector design where steel beams are made composite with the concrete deck. In both codes, studs are required to be designed for the fatigue and strength limit states. However, while both have the same basic requirements, the defining equations and procedures often differ. This design guide presents a procedural outline and example calculations for the design of stud shear connectors using LRFD. Part of its focus is comparing and contrasting LRFD with LFD.

The procedures and equations for each aspect of stud shear connector design are given below. The equations and code references shown refer to the AASHTO LRFD Code. Generally, if no commentary is provided, the equations or procedures are either identical or similar to the AASHTO LFD Code.

Provisions for curved steel girder design were merged with the existing provisions for straight girder design in the LRFD Code starting with the 2005 Interims. Many of the shear connector equations in the LRFD Code either refer to curved girders or have aspects embedded in them which are only intended for the design of studs for curved girders. This design guide focuses only on bridges with straight girders. As such, it also serves as a guide for simplifying and separating straight from curved girder LRFD provisions for the design of stud shear connectors.

LRFD Shear Connector Design, Procedure, Equations, and Outline

Determine Dead Load Contraflexure Points

When finding the dead load contraflexure points, use only the beam and slab (DC1), and superimposed dead loads (DC2). The weight of the future wearing surface (DW) should not be included. The dead load contraflexure points found using the AASHTO LRFD and LFD Specifications should be the same.

Determine Fatigue Loading

There are variances between the fatigue loading provisions for the AASHTO LRFD Code and the LFD Code. The LFD Code uses the standard HS20-44 live loading to determine shear and moment ranges used for fatigue design while LRFD does not use the standard HL-93 loading. Rather, Article 3.6.1.4 specifies a “fatigue truck”. This fatigue truck is similar to the truck portion of the HL-93 load, but has a constant 30 ft. rear axle spacing as opposed to a rear axle spacing which is variable. The LRFD fatigue loading also does not include the distributed lane load included in the HL-93 load.

When using the distribution factors contained in Section 4 of the LRFD Code for fatigue loading, the final distribution factor shall be divided by 1.2 to eliminate the single-lane multiple presence factor which is embedded in the equations (3.6.1.1.2).

For curved roadways on straight bridges, effects of centrifugal forces and superelevation (CE) are included in the LRFD fatigue loading (Table 3.4.1-1, Article 3.6.3). Dynamic load allowance or impact (IM) is also included and taken as 15% of the fatigue truck live load (Table 3.6.2.1-1). This value for impact is reduced from other load cases.

As specified in Table 3.4.1-1, loads used in LRFD fatigue loading shall be multiplied by a load factor of either 1.5 (Fatigue I, or infinite life fatigue loading), or 0.75 (Fatigue II, or finite life fatigue loading). Whether Fatigue I or Fatigue II loading is used is dependent upon the value of $(ADTT)_{SL}$. See “Find Required Shear Connector Pitch at Tenth-Points of Span” for more explanation.

Check Geometry

<u>Check Stud Dimensions</u>

The ratio of the height to the diameter of a stud shear connector shall not be less than 4.0 (6.10.10.1.1).

Stud shear connectors should penetrate at least two inches into the concrete deck (6.10.10.1.4). If fillets exceed 6 in., it is IDOT policy to reinforce the fillets to develop the shear studs. See Bridge Manual Section 3.3.9 for further guidance.

Clear cover for shear connectors shall not be less than two inches from the top of slab (6.10.10.1.4).

Calculate Number of Shear Connectors in Cross-Section (6.10.10.1.3)

Stud shear connectors shall not be closer than four stud diameters center-to-center across the top flange of a beam or girder. Article 6.10.10.1.3 requires the clear distance from the edge of the top flange to the edge of the nearest stud connector to be not less than 1 in. It is IDOT policy that the distance from the center of any stud to the edge of a beam shall not be less than 1 ½ in. ¾ in. ϕ studs, which are typically used, and ⅞ in. ϕ studs, which are occasionally used, provide more than the 1 in. clear distance to the edge of the flange required by LRFD. See also Bridge Manual Figure 3.3.9-1.

Find Required Shear Connector Pitch at Tenth-Points of Span

Calculate Pitch for Fatigue Limit State (6.10.10.1.2)

The required pitch, p (in.), of shear connectors shall satisfy:

$$p \leq \frac{nZ_r}{V_{sr}} \qquad \text{(Eq. 6.10.10.1.2-1)}$$

Where:

n = number of shear connectors in a cross-section

Z_r = fatigue shear resistance of an individual shear connector (kips). The value of Z_r is dependent upon the value of $(ADTT)_{75,\ SL}$, which is calculated as shown below:

$$(ADTT)_{75,\ SL} = p(ADTT_{75}) \qquad \text{(Eq. 3.6.1.4.2-1)}$$

Where:

p = percentage of truck traffic in a single lane in one direction, taken from Table 3.6.1.4.2-1.

$(ADTT)_{75,\,SL}$ is the projected amount of truck traffic at 75 years in a single lane in one direction, taken as a reduced percentage of the projected Average Daily Truck Traffic at 75 years ($ADTT_{75}$) for multiple lanes of travel in one direction.

Type, Size, and Location reports usually give ADTT in terms of present day and 20 years into the future. The ADTT at 75 years can be extrapolated from this data by assuming that the ADTT will grow at the same rate i.e. follow a straight-line extrapolation using the following formula:

$$ADTT_{75} = \left(\left(ADTT_{20} - ADTT_{0}\right)\left(\frac{75 \text{ years}}{20 \text{ years}}\right) + ADTT_{0} \right)(DD)$$

Where:

$ADTT_{20}$ = ADTT at 20 years in the future, given on TSL

$ADTT_{0}$ = present-day ADTT, given on TSL

DD = directional distribution, given on TSL

So, for example, if a bridge has a directional distribution of 50% / 50%, the ADTT for design should be the total volume divided by two. If the directional distribution of traffic was 70% / 30%, the ADTT for design should be the total volume times 0.7 in order to design for the beam experiencing the higher ADTT. If a bridge is one-directional, the ADTT for design is the full value, as the directional distribution equals one.

Once the value of $(ADTT)_{75,\,SL}$ is found, the value of Z_r is found as follows:

If $(ADTT)_{75,\,SL} > 960$ trucks/day, then

Z_r = $5.5d^2$ (Eq. 6.10.10.2-1)

and Fatigue I (infinite life) load combination is used. Otherwise,

Z_r = αd^2 (Eq. 6.10.10.2-2)

Where:

d = stud diameter (in.)

α = 34.5 – 4.28 log N (Eq. 6.10.10.2-2)

$$N = \left(365\,\frac{\text{days}}{\text{yr.}}\right)(75\text{ yrs.})\left(\frac{\text{no. cycles}}{\text{truck}}\right)(ADTT)_{37.5,\,SL}$$

(Eq. 6.6.1.2.5-2)

Where:

no.cycles/truck= number of stress cycles per truck passage, taken from Table 6.6.1.2.5-2

$(ADTT)_{37.5,\,SL}$ = single lane ADTT at 37.5 years. This is calculated in a similar fashion as the calculation of $(ADTT)_{75,\,SL}$ above except that the multiplier 37.5/20 is used in place of the multiplier 75/20 when extrapolating.

In the AASHTO LFD Code, at least 2 computations for fatigue strength are required when the ADTT is greater than or equal to 2500. One for N = 2,000,000 for multiple-lanes loaded, and one for N "over 2,000,000 cycles". This second computation assumes single-lane loading with a single-lane distribution factor. The objective of the LRFD Code is to determine a more accurate calculation of the number of fatigue stress cycles in comparison to LFD. This improved fatigue loading determination is used in conjunction with fatigue strength provisions that account for a number of cycles well beyond 2,000,000. The refinement in LRFD fatigue strength provisions from LFD is apparent in the formula used to calculate α which exists in both *specifications in different forms. Note, though, that Z_r calculated using LRFD is typically less than for LFD due to the large increase in the number of cycles being considered.*

V_{sr} = horizontal fatigue shear range per unit length (kip/in.)

$$= \sqrt{V_{fat}^2 + F_{fat}^2} \qquad \text{(Eq. 6.10.10.1.2-2)}$$

Where:

V_{fat} = longitudinal fatigue shear range per unit length (kip/in.)

$$= \frac{V_f Q}{I} \qquad \text{(Eq. 6.10.10.1.2-3)}$$

Where:

V_f = vertical shear force range under the fatigue load combination specified in Table 3.4.1-1 using the fatigue truck specified in Article 3.6.1.4 (kips).

Q = first moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section (in.^{3}).

I = moment of inertia of the short-term composite section (in.^{4})

F_{fat} = radial fatigue shear range per unit length (kip/in.)

For straight bridges with skews less than or equal to 45°, F_{fat} may be assumed to be zero throughout the entire bridge for most cases. The commentary of the 2006 interims in LRFD Article 6.10.10.1.2 suggests that radial fatigue analysis should be considered for all structures with skews greater than 20° and discontinuous cross-frames or diaphragms. Typical IDOT detailing practices for cross-frames and diaphragms is not considered discontinuous by the Department. See Section 3.3.5 of the Bridge Manual. As such, radial fatigue shear analysis is unnecessary for most typical bridges with skews less than or equal to 45°.

For straight bridges with skews greater than 45°, the Department recognizes that radial shear should be considered when designing stud shear connectors. Consequently, F_{fat} shall be examined in areas near diaphragms or cross-frames for these bridges, using the following equation:

$$F_{fat} = \quad F_{fat2} = \frac{F_{rc}}{w} \qquad \text{(Eq. 6.10.10.1.2-5)}$$

Where:

F_{rc} = net range of cross-frame or diaphragm force at the top flange (kips). In lieu of complex analysis, F_{rc} may be assumed to be 25 kips (See LRFD C6.10.10.1.2). The 25 kip load is a conservative estimate of the net range of cross-frame or diaphragm force at the top flange due to the fatigue truck loading.

w = effective length of deck over which F_{rc} is applied. This shall be taken as 24 in. for abutment lateral supports and 48 in. for all other staggered diaphragms or cross-frames.

F_{rc} should only be applied in composite areas within a length of w/2 on each side of cross-frames or diaphragms for bridges with skews greater than 45°.

AASHTO LFD Code does not mandate radial analysis for shear connectors design in straight bridges.

<u>Calculate Pitch for Strength Limit State</u> (6.10.10.4)

Calculate number of required connectors:

There are two equations for calculating the number of shear connectors required for strength. The first (Eq. 6.10.10.4.1-2) is used in end spans to calculate the number of connectors required for strength between a point of maximum positive moment and the exterior support. The second (Eq. 6.10.10.4.2-5) is used to calculate the number of connectors required for strength between interior supports and adjacent points of maximum positive moment.

$$n = \frac{P}{Q_r} \quad \text{(Eq. 6.10.10.4.1-2)}$$

Where:

n = total number of connectors required for strength

For sections between exterior supports and adjacent points of maximum positive moment:

$$P = \sqrt{P_p^2 + F_p^2} \quad \text{(Eq. 6.10.10.4.2-1)}$$

Where:

P_p = total longitudinal force in the concrete deck at the point of maximum positive live load moment (kips), taken as the lesser of:

$$P_{1p} = 0.85f'_c b_s t_s \quad \text{(Eq. 6.10.10.4.2-2)}$$

or

$$P_{2p} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \quad \text{(Eq. 6.10.10.4.2-3)}$$

Where:

b_{fc} = compression flange width (in.)

b_{ft} = tension flange width (in.)

b_s = effective flange width of composite section (in.)

D = web depth (in.)

t_s = slab thickness (in.)

t_{fc} = compression flange thickness (in.)

t_{ft} = tension flange thickness (in.)

t_w = web thickness (in.)

f'_c = concrete strength (ksi)

F_y = $F_{yw} = F_{yt} = F_{yc}$ = yield strength of steel (ksi)

Note that in wide flange sections and non-hybrid plate girders, Equation 6.10.10.4.2-3 simplifies to:

P_{2p} = $F_yA_{nc,}$ where A_{nc} is the total area of steel in the beam

For straight bridges, F_p, or radial shear force in the deck due to live load plus impact, shall be taken as zero (6.10.10.4.2).

For sections between interior supports and adjacent points of maximum positive moment:

$$P = \sqrt{P_T^2 + F_T^2} \qquad \text{(Eq. 6.10.10.4.2-5)}$$

Where:

P_T = total longitudinal force in the concrete deck between the point of maximum positive live load moment and centerline of an adjacent interior support (k). This is taken as the sum of the maximum possible force in the positive moment region (P_p, calculated above) and the maximum possible force in the negative moment region (P_n)

= $P_p + P_n$

P_p = see above (Eq. 6.10.10.4.2-2)

P_n is taken as the lesser of:

$$P_{1n} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \qquad \text{(Eq. 6.10.10.4.2-7)}$$

or

$$P_{2n} = 0.45f'_cb_st_s \qquad \text{(Eq. 6.10.10.4.2-8)}$$

Where all variables are as taken above. Note that for calculation of P_p, the steel section at the point of maximum positive moment must be used and for calculation of P_n, the steel section at the point of maximum negative moment must be used.

For straight bridges, F_T, or radial shear force in the deck due to live load plus impact, shall be taken as zero (6.10.10.4.2).

$$Q_r = \phi_{sc} Q_n \qquad \text{(Eq. 6.10.10.4.1-1)}$$

Where:

ϕ_{sc} = 0.85 (6.5.4.2)

Q_n = nominal resistance of one shear connector (kips)

$$= 0.5A_{sc}\sqrt{f'_c E_c} \le A_{sc}F_u \qquad \text{(Eq. 6.10.10.4.3-1)}$$

A_{sc} = cross-sectional area of one stud shear connector (in.^{2})

F_u = ultimate strength of stud (ksi) (6.4.4)

E_c = modulus of elasticity of concrete (ksi)

The pitch for the strength limit state is then found using the following equation:

$$p \le \frac{L}{n} \times \text{no. of connectors in cross-section}$$

Where:

L = length along beam from point of maximum positive moment to support (in.)

As per Article 6.10.10.1.2, the pitch shall not exceed 24 in. nor be less than six stud diameters, regardless of which is the controlling limit state.

Calculate Number of Additional Connectors at Permanent Load Contraflexure Points

As IDOT bridges are now designed composite even in negative moment regions, this requirement is no longer applicable.

It should be noted that shear studs shall be omitted from the tops of splice plates. As we are anticipating composite action over the splice plate even without any studs, simply detailing the studs to avoid the splice plate (rather than subtracting the splice plate from the length used to calculate strength requirements) is adequate.

LRFD Stud Shear Connector Design Example: Two-Span Wide Flange with 20° Skew

Design Stresses

f'_c = 3.5 ksi

f_y = 60 ksi (reinforcement)

F_y = $F_{yw} = F_{yt} = F_{yc}$ = 50 ksi (structural steel and stud shear connectors)

F_u = 60 ksi (stud shear connectors) (6.4.4)

$E_c = 33000 K_1 w_c^{1.5} \sqrt{f'_c}$ (Eq. 5.4.2.4-1)

Where:

K_1 = aggregate correction factor, normally taken as 1.0

w_c = weight of concrete (kcf). Normal weight of concrete is 0.150 kcf

$E_c = 33000(1.0)\left(0.150^{1.5}\right)\sqrt{3.5} = 3587$ ksi

Bridge Data

Span Length:	Two spans, symmetric, 98.75 ft. each
Bridge Roadway Width:	40 ft., stage construction, no pedestrian traffic
Slab Thickness t_s:	8 in.
Fillet Thickness:	Assume 0.75 in. for weight, do not use this area in the calculation of section properties
Future Wearing Surface:	50 psf
$ADTT_0$:	300 trucks
$ADTT_{20}$:	600 trucks

DD:	Two-Way Traffic (50% / 50%). Assume one lane each direction for fatigue loading
Number of Girders:	6
Girder Spacing:	7.25 ft., non-flared, all beam spacings equal
Overhang Length:	3.542 ft.
Skew:	20°

Non-Composite Section Data

The following are the flange and web sections for the positive moment region:

$D = 42$ in.

$t_w = 0.4375$ in.

$b_{tf} = b_{bf} = 12$ in.

$t_{bf} = 0.875$ in.

$t_{tf} = 0.75$ in.

The following are the flange and web sections for the negative moment region:

$D = 42$ in.

$t_w = 0.5$ in.

$b_{bf} = b_{tf} = 12$ in.

$t_{bf} = 2.5$ in.

$t_{tf} = 2.0$ in.

The points of dead load contraflexure has been determined to be approximately 67 ft. into span one and 31.75 ft. into span two. Section changes will occur at these points.

Composite Section Data

Effective Flange Width = 87 in.

Y_b = 24.3 in. (assuming a ½ in. “non-structural” fillet)

I = 31067 in.^{4} for positive moment region, 63119 in.^{4} for negative moment region

Q = 696 in.^{3} for positive moment region, 1127 in.^{3} for negative moment region

Dead Load Contraflexure Points

67 ft. from abutment bearings (0.68 $\times$ span 1, 0.32 $\times$ span 2).

Fatigue Load Combination Shears at Tenth-Points of Bridge

Point	$V^{+}_{FATIGUE\ I}$(kips)	$V^{-}_{FATIGUE\ I}$(kips)	$V^{+}_{FATIGUE\ II}$(kips)	$V^{-}_{FATIGUE\ II}$(kips)
0.0	55.0	-7.5	27.5	-3.8
0.1	42.9	-7.0	21.5	-3.5
0.2	35.1	-7.9	17.5	-4.0
0.3	27.7	-11.7	13.9	-5.8
0.4	21.1	-19.1	10.6	-9.5
0.5	15.1	-26.9	7.6	-13.4
0.6	9.8	-34.6	4.9	-17.3
0.7	5.3	-41.7	2.7	-20.8
0.8	1.6	-48.1	0.8	-24.1
0.9	0.0	-53.9	0.0	-27.0
1.0	0.0	-58.9	0.0	-29.5

As the bridge is symmetric, only span one is shown. Span two is similar by rotation.

Check Geometry

Check Stud Dimensions:

Using 4 in. long, ¾ in. ϕ studs, the ratio of height to diameter is

$$\frac{h}{d} = \frac{4\text{ in.}}{0.75\text{ in.}} = 5.33 > 4 \qquad \text{OK} \qquad (6.10.10.1.1)$$

Fillets for this bridge do not exceed 2 inches, therefore the stud shear connectors penetrate at least two inches into the slab. The studs are not long enough to extend within two inches of the top of slab (6.10.10.1.4).

Calculate Number of Shear Connectors in Cross-Section: (6.10.10.1.3)

The flange width is 12 in. Placing the studs a minimum distance of four diameters apart center-to-center with a center-of-stud to edge-of-flange distance of 1 ½ in. allows three studs per row.

Find Required Shear Connector Pitch at Tenth-Points of Span

Calculate Pitch for Fatigue Limit State (6.10.10.1.2)

$$p \leq \frac{nZ_r}{V_{sr}}$$ (Eq. 6.10.10.1.2-1)

Where:

n = 3 studs per row

$(ADTT)_{75,\,SL}$ = $p(ADTT)$ (Eq. 3.6.1.4.2-1)

Where:

$$ADTT = \left(\left(600\,\frac{\text{trucks}}{\text{day}} - 300\,\frac{\text{trucks}}{\text{day}}\right)\left(\frac{75\text{ years}}{20\text{ years}}\right) + 300\,\frac{\text{trucks}}{\text{day}}\right)(0.5)$$

= 713 trucks/day

p = 1.0 for one lane (not counting shoulders) (Table 3.6.1.4.2-1)

$(ADTT)_{75,SL}$ = 1.0(713 trucks/day) = 713 trucks/day < 960 trucks/day. Use Fatigue II load combination.

$Z_r = \alpha d^2$ (Eq. 6.10.10.2-2)

Where:

α = 34.5 – 4.28 log N

$$N = \left(\frac{365 \text{ days}}{\text{year}}\right)(75 \text{ years})\left(\frac{n \text{ cycles}}{\text{truck}}\right)\left(\frac{(ADTT)_{37.5,\,SL}\ \text{trucks}}{\text{day}}\right)$$

(Eq. 6.6.1.2.5-2)

For points 0.0-0.9:

n = 1.0 (Table 6.6.1.2.5-2)

$$(ADTT)_{37.5,\,SL} = \left(\left(600\,\frac{\text{trucks}}{\text{day}} - 300\,\frac{\text{trucks}}{\text{day}}\right)\left(\frac{37.5 \text{ years}}{20 \text{ years}}\right) + 300\,\frac{\text{trucks}}{\text{day}}\right)(0.5)$$

= 431 trucks/day

$$N = \left(\frac{365 \text{ days}}{\text{year}}\right)(75 \text{ years})\left(\frac{1 \text{ cycle}}{\text{truck}}\right)\left(\frac{431 \text{ trucks}}{\text{day}}\right)$$

= 11.8×10^6 cycles

α = 34.5 – 4.28 log 11.8×10^6

= 4.23

Z_r = $4.23(0.75)^2$

= 2.38 k

For points 0.9-1.0:

n = 1.5 (Table 6.6.1.2.5-2)

$$(ADTT)_{37.5,\,SL} = \left(\left(600\,\frac{\text{trucks}}{\text{day}} - 300\,\frac{\text{trucks}}{\text{day}}\right)\left(\frac{37.5 \text{ years}}{20 \text{ years}}\right) + 300\,\frac{\text{trucks}}{\text{day}}\right)(0.5)$$

= 431 trucks/day

$$N = \left(\frac{365 \text{ days}}{\text{year}}\right)(75 \text{ years})\left(\frac{1.5 \text{ cycle}}{\text{truck}}\right)\left(\frac{431 \text{ trucks}}{\text{day}}\right)$$

= 17.7×10^6 cycles

$$\alpha = 34.5 - 4.28 \log 17.7 \times 10^6$$
$$= 3.48$$

$$Z_r = 3.48(0.75)^2$$
$$= 1.96 \text{ k}$$

$$V_{sr} = \sqrt{V_{fat}^2 + F_{fat}^2}$$

Where:

$$V_{fat} = \frac{V_f Q}{I}$$

For positive moment regions (points 0.0 to 0.68):

$Q = 696 \text{ in.}^3$

$I = 31067 \text{ in.}^4$

For negative moment regions (points 0.0 to 0.68):

$Q = 1127 \text{ in.}^3$

$I = 63119 \text{ in.}^4$

At each tenth point within the positive moment region, V_{fat} is calculated from the shear range V_f as:

Point	V_f (kips)	Q (in.^{3})	I (in.^{4})	V_{fat} (k/in.)
0.0	31.3	696	31067	0.70
0.1	25.0	696	31067	0.56
0.2	21.5	696	31067	0.48
0.3	19.7	696	31067	0.44
0.4	20.1	696	31067	0.45
0.5	21.0	696	31067	0.47
0.6	22.8	696	31067	0.51

0.7	23.5	1127	63119	0.42
0.8	24.9	1127	63119	0.44
0.9	27.0	1127	63119	0.48
1.0	29.5	1127	63119	0.53

F_{fat} = 0 kips (diaphragms are considered continuous and skew ≤ 45°). Therefore, V_{sr} = V_{fat}.

The resulting maximum pitches for Fatigue Limit State, $\frac{nZ_r}{V_{sr}}$, are:

Point	n	Z_r (k)	V_{sr} (k/in.)	p (in.)
0.0	3	2.38	0.70	10.2
0.1	3	2.38	0.56	12.8
0.2	3	2.38	0.48	14.9
0.3	3	2.38	0.44	16.2
0.4	3	2.38	0.45	15.9
0.5	3	2.38	0.47	15.2
0.6	3	2.38	0.51	14.0
0.7	3	2.38	0.42	17.0
0.8	3	2.38	0.44	16.2
0.9	3	1.96	0.48	12.3
1.0	3	1.96	0.53	11.1

Calculate Pitch for Strength Limit State (6.10.10.4)

Calculate number of required connectors:

$$n = \frac{P}{Q_r} \quad \text{(Eq. 6.10.10.4.1-2)}$$

For region from abutment to maximum positive moment:

$$P = \sqrt{P_p^2 + F_p^2} \quad \text{(Eq. 6.10.10.4.2-1)}$$

Where:

$P_{1p} = 0.85f'_c b_s t_s$ (Eq. 6.10.10.4.2-2)

or

$P_{2p} = F_y A_{nc}$ (Simplified Eq. 6.10.10.4.2-3)

Where:

f'_c = 3.5 ksi

b_s = 87 in.

t_s = 8 in.

F_y = 50 ksi

A_{nc} = 37.875 in.^{2}

$P_{1p} = 0.85(3.5 \text{ ksi})(87 \text{ in.})(8 \text{ in.}) = 2070.6 \text{ k}$

$P_{2p} = (50 \text{ ksi})(37.875 \text{ in.}^2) = 1893.75 \text{ k}$

$\therefore P_{2p}$ controls

F_p = 0 kips

P = 1893.75 k

For region from maximum positive moment to pier:

$P = \sqrt{P_T^2 + F_T^2}$ (Eq. 6.10.10.4.2-1)

Where:

$P_T = P_p + P_n$

P_p = 1893.75 k

P_n = lesser of P_{1n} and P_{2n}

$P_{1n} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc}$ (Eq. 6.10.10.4.2-7)

$P_{2n} = 0.45f'_c b_s t_s$ (Eq. 6.10.10.4.2-8)

Where:

f'_c = 3.5 ksi

b_s = 87 in.

t_s = 8 in.

F_y = 50 ksi

A_{nc} = 75 in.^{2}

P_{1n} = (50 ksi)(75 in.^{2}) = 3750 kips

P_{2n} = 0.45(3.5 ksi)(87 in.)(8 in.) = 1096.2 kips

$\therefore P_{2n}$ controls

F_p = 0 kips

P = 1893.75 k + 1096.2 k

= 2989.95 k

$Q_r = \phi_{sc}Q_n$ (Eq. 6.10.10.4.1-1)

Where:

ϕ_{sc} = 0.85 (6.5.4.2)

$Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq A_{sc}F_u$ (Eq. 6.10.10.4.3-1)

Where:

$A_{sc} = \pi(0.375\text{ in.})^2 = 0.44\text{ in.}^2$

f'_c = 3.5 ksi

E_c = 3587 ksi

F_u = 60 ksi (6.4.4)

$$0.5A_{sc}\sqrt{f'_c E_c} = 0.5(0.44 \text{ in.}^2)\sqrt{(3.5 \text{ ksi})(3587 \text{ ksi})} = 24.7 \text{ kips}$$

$$A_{sc}F_u = (0.44 \text{ in.}^2)(60 \text{ ksi}) = 26.4 \text{ kips}$$

$$\therefore Q_n = 24.7 \text{ kips}$$

And:

$$Q_r = \phi_{sc} Q_n = 0.85(24.7) = 21.0 \text{ kips}$$

$$n = \frac{1893.75 \text{ k}}{21.0 \text{ k}} = 90.2 \text{ studs required from abutment to point of maximum positive moment}$$

$$n = \frac{2989.95 \text{ k}}{21.0 \text{ k}} = 142.4 \text{ studs required from point of maximum positive moment to interior support}$$

Assuming the point of maximum moment occurs at 0.375 of the span length, the required pitch for strength limit state is:

$$p \le \frac{(0.375 - 0.0)(98.75 \text{ ft.})\left(12\frac{\text{in.}}{\text{ft.}}\right)\left(3\frac{\text{studs}}{\text{row}}\right)}{90.2 \text{ studs}} = 14.8\frac{\text{in.}}{\text{row}} \quad \text{for points 0.0 to 0.375}$$

$$p \le \frac{(1.0 - 0.375)(98.75 \text{ ft.})\left(12\frac{\text{in.}}{\text{ft.}}\right)\left(3\frac{\text{studs}}{\text{row}}\right)}{142.4 \text{ studs}} = 15.6\frac{\text{in.}}{\text{row}} \quad \text{for points 0.375 to 1.0}$$

Design Summary

The controlling pitches for design are as follows:

Point	p (fatigue) (in.)	p (strength) (in.)	controlling p (in.)
0.0	10.2	14.8	10.2
0.1	12.8	14.8	12.8
0.2	14.9	14.8	14.8
0.3	16.2	14.8	14.8
0.4	15.9	15.6	15.6
0.5	15.2	15.6	15.2
0.6	14.0	15.6	14.0
0.7	17.0	15.6	15.6
0.8	16.2	15.6	15.6
0.9	12.3	15.6	12.3
1.0	11.1	15.6	11.1